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MORNING-GLORY SHAFT SPILLWAYS: PERFORMANCE TESTS ON PROTOTYPE AND MODEL

A. J. Peterka,[1] A.M. ASCE

Introduction

This paper compares the performance of the Heart Butte Dam morning-glory spillway and outlet works model with the performance of the prototype structure, and also describes certain elements of the prototype performance which could not be included in the model tests. The results of the comparison add further proof to the premise that prototype performance can be predicted with accuracy from model tests.

There is a general need for data which can be used to compare the performance of models and prototypes and extend the range of usefulness of models as an aid in design. Prototype data are often difficult to obtain and then usually the model data are not in the prototype range of heads or discharges, making it difficult to make direct comparisons. The Heart Butte spillway, however, operated during the first flood season following its completion and almost immediately after the hydraulic model tests were made. With the model test data still fresh it was possible to obtain prototype data on short notice that could be compared with model tests.

A brief discussion is given of the hydraulic model tests conducted on a 1:21.5 scale model to aid in the design of the structure and to obtain data useful in operating the prototype structure. Following a description of the 1950 flood on the Heart River which produced a discharge of 68 percent of the maximum anticipated outflow, the performance of the prototype structure is described.

Direct model-prototype comparisons are made of spillway performance and discharge for free and submerged conditions; spillway air demand; stilling basin performance; including erosion downstream from the basin; and tailwater elevations in the excavated channel. Photographs and charts are used to illustrate the agreement found between model and prototype performance.[2]

Certain aspects of the prototype performance which are beyond the scope of model tests are also discussed, including the effect of ice completely covering the morning-glory during submerged discharge, the erosion of the downstream riverbanks, and the effectiveness of the riprap used on the excavated channel banks. The results of an inspection of the spillway tunnel and structure following the 1950 and 1951 floods are also given.

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2. Please note that Figures 7 and 14-17 inclusive have been eliminated by the author.

Description Of Project

The Heart Butte Dam is located on the Heart River 60 miles west of Bismarck, North Dakota, and is a part of the Heart River Unit of the Missouri River Basin Project, Figure 1. The dam is of compacted earth fill with a rock riprap cover, rises 135 feet above the Heart River stream bed, and is 1,860 feet long, Figure 2. The dam is a combined irrigation and flood control structure, with no power being developed. The reservoir at maximum water-surface elevation will contain 392,500 acre-feet from a drainage area of 1,810 square miles.

The flood control spillway, located near the right abutment, consists of a morning-glory spillway having an outside diameter of 32 feet 6 inches, discharging into a vertical shaft 11 feet in diameter. The shaft is connected to a 90° vertical bend and nearly horizontal tunnel 14 feet in diameter and about 800 feet long, Figure 3, which leads to the hydraulic jump stilling basin, Figure 2. The maximum vertical fall from headwater to stilling basin floor is about 130 feet.

The morning-glory spillway is unusual in that it is designed to operate throughout the range of free discharge, throughout the transition range between free and submerged discharge, and up to submergence as great as 53.7 feet of water above the crest. The spillway crest is equipped with six equally spaced piers placed radially in plan, but does not have control gates of any kind. The outlet works used primarily for release of irrigation water, Figures 2 and 3, is an integral part of the spillway structure. The entrance to the outlet works encircles the vertical shaft of the spillway and discharges into a 5-foot 3-inch-diameter tunnel located directly above the spillway tunnel. The smaller tunnel is controlled at its lower end by a 4- by 5-foot, high-pressure slide gate and discharges into the spillway tunnel, entering the larger tunnel from above through a specially designed junction section. The spillway tunnel then carries the outlet works discharge into the single stilling basin used for both spillway and outlet works discharges, Figure 2.

The capacity of the outlet works is 650 second feet with reservoir elevation at spillway crest level, elevation 2064.50. For normal operation, the outlet works will be closed when the spillway is in operation. The capacity of the spillway is 5,450 second feet at maximum reservoir elevation 2118.2.

The feasibility of a combined spillway and outlet structure was determined, and the detailed shape and arrangement of the various parts of the structure were developed from hydraulic model tests on a 1:21.5 scale model.

Summary Of Hydraulic Model Tests

The Model

Tests were made on a 1:21.5 scale model of the discharge structures, including the morning-glory, the outlet works intake structure, and the surrounding topography which were constructed within the head box; the two tunnels including the outlet works control gate, the 90° vertical bend, and tunnel junction section, which were built outside of the head box; the stilling basin common to both the spillway and outlet works, and a portion of the down river topography, which were constructed within the tail box. Much of the structure was modeled in transparent plastic to permit the observation of flow conditions throughout the structure, Figure 3.

Spillway and Pier Tests

Tests on a preliminary design of the morning-glory spillway indicated that the discharge capacity was larger than necessary. Consequently, the vertical shaft diameter was reduced from 14 to 11 feet, the spillway profile was reshaped to fit the vertical shaft, and a 90° vertical transition bend was installed. The discharge capacity was then found to be approximately correct, according to the irrigation and flood control requirements.

Vortices which formed in the model when the spillway was submerged, Figure 4A, were thoroughly investigated, experimentally and mathematically, and when it was found that these same vortices could form to scale in the prototype, attempts were made to eliminate them. Various arrangements of piers, dividing walls, and floating and fixed rafts were tested, and as a result, six spillway crest piers were recommended for use on the prototype, see Figure 5. It was found unnecessary to extend the piers as high as the maximum headwater elevation, a distance of 54 feet. Since vortex action diminished rapidly when the head on the crest approached 14 feet, it was necessary to extend the piers only to this height.

Deflector and Vertical Bend Tests

The tests to determine the most satisfactory type of vertical bend showed that a diverging elbow joining the 11-foot-diameter shaft with the 14-foot-diameter horizontal tunnel had a distinct advantage in that it provided greater space between the water surface and the tunnel crown for ventilation in the vertical bend from the atmosphere at the tunnel outlet.

However, with this arrangement in place, difficulty was encountered in preventing the horizontal tunnel from filling unexpectedly when the spillway and outlet works were both operating. Flow passing through the bend did not break cleanly from the crown of the bend. The flow had a tendency to follow the crown throughout the bend, causing a change in the location of the flow control. When the control moved downstream the head on the system increased, causing a temporary increase in discharge which filled the tunnel. This resulted in negative pressures of considerable magnitude occurring on the spillway face. Once the tunnel filled, it was impossible to obtain open channel flow again unless the head on the spillway was reduced considerably below the point where it had filled. Consequently, a small deflector was placed at the base of the vertical shaft on the downstream, or crown, side of the shaft, see Figure 5. The deflector accomplished three things: (1) it provided a positive control at the base of the vertical shaft and prevented the tunnel from filling, (2) it had a stabilizing effect on smaller flows and provided a flat water surface on all flows passing into the vertical bend, and (3) it provided a clear passage for air to circulate as far upstream as the base of the deflector. The thickness of the deflector at the base was varied in the model to determine the size necessary to exactly meet the discharge requirements at certain heads since precise tests had shown that the 11-foot-diameter vertical shaft was slightly too large. Spillway flow in the tunnel was found to be satisfactory after the structure had been modified as described. Figure 6 shows the flow entering, passing through, and leaving the vertical bend with the deflector in place. Note the smooth and flat water surface on the flow entering the tunnel.

Stilling Basin Tests

An effective energy-dissipating device was required in the stilling basin because of the friable nature of the material in the river channel and riverbanks. Even moderate erosion tendencies and wave heights could not be tolerated. Consequently, it was felt that a hydraulic jump basin would be necessary to provide good energy dissipation and a smooth water surface in the downstream channel. The first stilling basin tests indicated that the main problem was concerned with spreading the high-velocity water, about 60 feet per second, into a uniformly distributed sheet suitable for the formation of a jump. The first attempt to induce lateral spreading was by means of a sudden rise in the stilling basin floor downstream from the tunnel portal. It was found that a hump sufficiently long to produce even a moderate amount of spreading resulted in an extremely long stilling basin structure. With a basin of reasonable length, sufficient spreading could not be produced to permit the formation of an effective jump. The problem was solved by discharging the flow onto a horizontal floor about 23 feet long after it had passed through a transition section at the end of the tunnel which started the spreading of the flow, see Figure 8. The flat floor induced more spreading before the flow dropped downward on the trajectory curve. Tests showed that this arrangement produced good lateral distribution of flow as far downstream as the trajectory curve and fairly good distribution beyond this point. The addition of two low walls, placed so as to divide the basin approximately into thirds, produced excellent downstream distribution of flow and an efficient hydraulic jump in the basin. The walls, which varied from 3 to 4 feet high throughout their length, did not extend upward through the flow for high discharges but produced the desired effect of distributing the flow from 14 feet wide at the tunnel portal to an ultimate 42.5 feet wide in a horizontal distance of 75 feet.

Chute blocks and baffle piers were used to increase the fine grain turbulence in the basin and thereby reduce the required length of the stilling basin. The shape of the baffle piers, dividing wall noses, and trajectory curves were modified to provide atmospheric pressures or above on critical areas, since tests on preliminary designs had indicated that pressures as low as 18 feet of water below atmospheric pressure occurred downstream from sharp corners. The recommended stilling basin is shown in Figure 8.

The performance of the developed stilling basin was evaluated from erosion tests made on a movable bed located downstream from the model basin and from wave height observations made in the excavated tailrace channel. Erosion tests were made using a well-graded sand (100 percent passed a No. 4 sieve and 3 percent passed a No. 50 sieve). These tests showed that erosion tendencies were less severe on the channel bottom than on the sloping banks. Wave action originating in the hydraulic jump combined with a slight surging action caused rapid decay of the banks. Every effort was made to keep the waves and surges to a minimum, but it was deemed necessary to riprap the banks of the prototype. Figure 9 shows the performance of the recommended basin.

Spillway Air Tests

When the morning-glory spillway was designed, it was believed that air introduced into the spillway discharge at a point just below the spillway crest might help to cushion the impact of the flow passing around the vertical bend. It was important that unnecessary impact and vibrations caused by the flowing water be eliminated, because the entire structure was to be constructed on sand. Futhermore, if for any reason cavitation should occur in or near the

vertical bend, the presence of the entrained air might reduce the tendency to damage the concrete tunnel lining. Laboratory tests have shown that even very small quantities of air introduced into the flow will delay the appearance of cavitation damage.[3]

Model tests on the many devices proposed to increase the entrained air in the flow showed that only a relatively small amount of air entered the flow regardless of how the air-entraining devices were arranged. However, it was known that air flow in small hydraulic models is uncertain and that a greater percentage of air can be expected to enter a similar prototype structure. The amount of increase to be expected in the prototype is not known and cannot be computed since the factors governing the entrainment of air are not known. After tests on many different model arrangements, it was finally decided to construct the prototype air vents shown in Figure 5 and to provide measuring facilities in the prototype structure so that air quantity determinations could be made. Figure 6 shows the vertical bend discharging 3,750 cfs with air, induced by the air deflectors, entrained in the flow. To the unaided eye the air flow appeared continuous but in the 1/15,000-second exposure photograph the air is shown to enter in gusts. This is more clearly illustrated in the extremely slow motion pictures made of this condition.

Description Of 1950 Spring Flood

Preceding the heavy run-off in April 1950, the weather had been cold and the ground was frozen and covered with snow. A stiff wind had blown the snow off the ridges, concentrating it on the slopes and in the valleys of the drainage area. The weather then turned unseasonably warm, causing a fast melt and heavy run-off from the frozen terrain. On April 15, 1950, the temperature was about 80°, and the snow melt caused an increase in the inflow to the reservoir from 5,000 to 30,500 cfs on April 16, see Figure 10. The high run-off and inflow continued throughout April 17 and most of April 18. The spillway went into operation on April 17, reaching a peak flow of 3,760 second feet on April 19 and continued without appreciable reduction in discharge through April 29, a period of over 12 days. The maximum outflow discharge represented 68 percent of the anticipated maximum outflow, and the maximum reservoir elevation indicated that 38 percent of the flood storage had been utilized. Figure 11 shows the hydraulic data in terms of the spillway elevations. At the time of maximum outflow, the spillway crest was submerged 17.24 feet, making the reservoir elevation 3.24 feet over the tops of the spillway piers, see Figure 11. The maximum height of fall, headwater to tail water, was 72 feet, and the energy entering the stilling basin amounted to 31,000 horsepower.

The Heart River, on which Heart Butte Dam is constructed, flows into the Missouri River at Mandan, North Dakota, about 6 miles from Bismarck, Figure 1. Some flood damage occurred at Mandan, caused primarily by high water in the Missouri River. Both rail and highway travel were impossible during the high water. The Heart Butte Dam undoubtedly reduced the flood crest at Mandan, but no figures are available as to extent. The structure operated as intended and therefore provided as much protection as was anticipated.

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3. The Effect of Entrained Air on Cavitation Pitting - A. J. Peterka - Proceedings, Minnesota International Hydraulics Convention - ASCE, IAHR, 1953.

Model-Prototype Comparison Tests

It was recognized that model-prototype comparison data pertaining to the spillway discharge and the air demand would be particularly valuable and that comparisons of the erosion in the excavated channel, wave heights below the stilling basin, and profiles below the stilling basin would also be of interest. In the course of recording these data, other comparisons were made which included observations on vortex formation above the spillway and a comparison of the computed and actual tail water curves in the excavated channel and in the river. Water-surface profiles in the stilling basin and data on the riprap protection were also obtained.

Spillway Capacity

During the 1950 run-off, when the headwater was above the spillway crest, readings were taken each morning and afternoon on the headwater gage located in the gate operating house. These are shown plotted in Figure 11. Using the discharge-capacity curve obtained from the model tests on the morning-glory spillway, Figure 12, an outflow hydrograph was prepared, see Figure 13. On April 17, 19, 25, and May 1 the United States Geological Survey made stream gage measurements in the river downstream from the stilling basin to determine the discharge of the spillway. During these measurements, the irrigation outlet works was closed. The discharges determined by the United States Geological Survey, indicated by circles on Figure 13, indicate the degree of agreement between the model and the prototype measurements. Differences were 4.6, 1.1, and 1.8 percent for the April 17, 19, and 25 determinations, respectively. For all practical purposes, these points indicate good agreement between model and prototype discharge characteristics. On May 1 the difference was 23.4 percent, indicating considerable disagreement. The measurements on April 17 and May 1 were not made under ideal conditions. The United States Geological Survey notes for April 17 indicated that ice in the channel may have affected the measurements, and on May 1, when the greatest disagreement was found, that a wind was blowing which might have altered the relation between the head on the crest and the headwater gage reading. Another possible cause for the discrepancy might be the rapidly falling stage in the reservoir during the measurements on May 1 as indicated in the hydrograph of Figure 13. In general, however, the agreement between model and prototype discharges is considered excellent, particularly at the higher discharges, and it is believed that the rating curve obtained from the model will adequately serve to determine discharge values through the prototype morning-glory spillway.

Spillway Performance--Free and Submerged Discharges

During the model tests it was noticed that for certain arrangements of the structure the transition from free to submerged flow, and vice versa, was accompanied by violent surging in the vertical shaft. In some cases the unstable flow condition existed over several feet of change in reservoir elevation. A mushroom-shaped column of water rose and fell in the shaft, causing excessive splashing and turbulence. In addition to giving poor hydraulic conditions it was feared that the prototype structure would be subjected to objectionable forces and vibration. Consequently, the structure recommended for field construction was developed by model tests to provide a minimum transition range, i.e., less than 0.2 foot prototype. The rating curve determined by model tests, see Figure 12, indicates the definite change from one type of flow to the other.

It was for this reason that the prototype spillway was closely observed when the headwater reached the transition range.

On April 16, 1950, the reservoir had risen to the spillway crest elevation 2064.5. Ice covered most of the reservoir area, but there was some open water close to the spillway. By April 17, 1950, the reservoir had risen sufficiently to submerge the spillway and provide a head of 9.3 feet on the crest, corresponding to a discharge of 3,250 cfs. Sometime during the night the reservoir elevation had passed through the critical region where the flow changes from free to submerged. Some ice had been discharged through the spillway, but it had caused no apparent difficulty. On April 18, the piers were completely covered and the reservoir was completely covered with ice which appeared to be about 12 inches thick. A small amount of trash had collected over the spillway and slight movement of the trash was the only evidence that the spillway existed. The reservoir continued to rise throughout April 19, but on April 20 it started to recede. On April 21 the reservoir was still a foot or so above the piers. The ice was breaking up fast and the wind was shifting it around the spillway area. Regardless of whether ice or water was over the spillway entrance, the operation was satisfactory, with no evidence of serious vortex action.

On April 26, with the reservoir at elevation 2074 and the piers again visible, operation was also satisfactory. Water inside the pier structure was level with the reservoir and only occasional light turbulence was visible.

On April 28, the reservoir was down to elevation 2071, or about 0.7 foot above the point where the flow changes from submerged to free discharge, see Figures 18 and 12. The photograph indicates the mild condition inside the spillway. There was no pulsation or rising and falling of the "mushroom."

On April 29, the reservoir had fallen to 0.8 foot below the critical submergence point and although the "mushroom" was lower in the shaft, it was still stable with no rising and falling evident. Again the flow had passed through the critical range during the night when photographs and observations were impracticable. Indications are that the prototype submerged at about the headwater elevation shown by the break on the curve of Figure 12 and that the change occurred abruptly as indicated on the model curve.

On April 30, the spillway was discharging freely (1,600 cfs) with reservoir elevation 2068.6, see Figure 19. No spray emerged from the glory hole at this or lower heads as has been noted on some other glory hole spillways.

Throughout the flow range there was no vibration noticeable in the structure. Several excursions down into the outlet works gate access well were made while the spillway was operating. Efforts were made to detect vibration in the structure by feeling the various parts of the structure, but no vibration could be detected. Also, there was no "noise" from the spillway at any head that could be detected from the top of the dam or from the reservoir banks. The outlet works gate was opened and closed on April 21, 1950. No noise or vibration was evident during this operation.

One year later the spillway again went into operation, reaching a maximum reservoir elevation of 2075, or about 7 feet less than occurred in 1950. On March 27, 1951, the reservoir was at elevation 2070, see Figure 20, about 0.2 foot below the submergence point. Again the operation was satisfactory with no visible difficulty despite the fact that on February 15, 1951, the ice in the reservoir was 36 inches thick. There was no difficulty due to ice.

Spillway Air Demand

Measurements were made in the model to determine the quantity of air being entrained by the spillway discharge as it passed over the air-entraining deflectors located on the spillway face just below the spillway crest, see Figure 5. Air-flow measurements in the model were made using a 3/8-inch-diameter sharp-edged orifice connected to a differential water manometer. All air entering the model passed through the orifice before entering the venting system. Since the differential was extremely small for the air quantity flowing in the model, a specially constructed gage was used which multiplied the actual differential so that more consistent readings could be obtained throughout a series of tests. The gage was calibrated to provide reasonably accurate air measurements, but consistency was considered more important than absolute accuracy.

At the time of prototype construction, pipe was extremely difficult to obtain on short notice. Since the model tests continued throughout most of the construction period, only a small amount of pipe and special fittings could be provided for measuring stations in the prototype. Thus, the data obtained from the prototype are not sufficient to determine pressures in various parts of the venting system, but do indicate the quantity of air flow in the prototype for various spillway discharges.

The air quantity flowing in the prototype vents was determined by measuring the air velocity with an anemometer hand-held in the 18-inch-diameter air vent pipes. Air-velocity determinations were made in one of the vertical pipes contained in the wall of the gate operating house and in one of the horizontal pipes just upstream from the point where it emerges into the tunnel junction section, see Figure 21. Concurrent with the air-velocity measurements, pressure measurements were made in the other horizontal air vent using a U-tube water manometer. The pressure-measuring station is also shown in Figure 21.

Air flow in the prototype was not smooth, as evidenced by the sound of the air flow and from the difficulty experienced in holding the anemometer steady. There was chance for considerable error in any one anemometer measurement and so several determinations were made for each flow in both the vertical and horizontal vent pipes. Readings were taken until the observer was satisfied that a true average had been obtained; a consistent set of readings over a period of up to 12 minutes was obtained. The anemometer recorded lineal feet of flow which, when divided by the elapsed time, gave the air velocity in feet per second. Each observation lasted about 2 minutes so that the average velocity of air flow was that occurring for a testing time of 6 to 12 minutes. Pressures measured in the U-tube also indicated that the air flow was not steady. Differentials varied from plus to minus, but average readings were relatively easier to obtain.

The results of the air quantity and pressure determinations are plotted on Figure 22. The percentage of air entrained in the spillway discharge, for both model and prototype, showed a decrease as the discharge increased. In this respect the model predicted the performance of the prototype. The prototype, however, entrained roughly four times as much air as was predicted by the model. In this respect, also, the prototype performed as anticipated except that accurate predictions could not be made from the model tests to determine how much more air the prototype would entrain. Where the model showed air entrainment of 5.5 percent of the water discharge for 1,000 second feet of spillway discharge, the prototype showed 20.5 percent. For 3,600 second feet the model showed 1.9 percent and the prototype 7.7 percent.

The points from which the curves of Figure 22 were drawn are also shown in the figure. The prototype air demand curve was not drawn through the points for 2,500 second feet because the pressure values, which were considered more reliable, indicated that the curve should be drawn below the velocity points. Moreover, the shape of the curve was then similar to the model curve which was based on very consistent data. To further prove the validity of the shape and values of the prototype air demand curve, computations of air flow were made using the measured pressures, assuming that both vent pipes carried equal quantities of air and using the usual losses for bends, friction, inlet, etc. The computed values were found to be in fair agreement with the curve values.

Performance of the Stilling Basin

The performance of the stilling basin was satisfactory in every respect and, furthermore, it performed according to the predictions made from the model tests. A general view of the basin and surrounding area is shown in Figure 23.

Water leaving the tunnel appeared to be well aerated and at the approximate depth indicated in the model studies. The entire basin contained extremely turbulent water, see Figure 24, and was long enough to obtain the full jump height before the flow entered the excavated channel, see Figure 25. A considerable amount of spray was thrown into the air, at times, where the outflow from the tunnel plunged beneath the tail water, but most of the spray fell back into the basin. The small amount of spray which fell adjacent to the basin caused no difficulty. Much of the time the flow entered the basin smoothly. Flow leaving the basin had a relatively quiet water surface with few measurable waves, Figure 25. There were long-period swells, however, with a maximum height of 12 to 18 inches which were caused by pulsations set up in the stilling basin. The disturbances below the stilling basin were similar to those noted during the model tests.

Water-surface profiles were measured in the prototype for discharges of 3,700, 3,300, 2,350, and 1,050 cfs. These are shown in Figures 26 and 27, along with the profile obtained during the model tests for 5,600 cfs. Although no exact comparisons can be made, the prototype profiles seem to be in good agreement with the model profile. If differences do exist, they are probably due to the greater air entrainment in the prototype, making the prototype profiles slightly higher than those in the model for the same discharge.

Erosion Downstream from Stilling Basin

Erosion tests in the model had indicated that the channel banks just downstream from the stilling basin would be subjected to greater erosion forces than the channel bottom and that rock riprap would be necessary in the prototype to prevent bank damage. The channel bottom was shown by the model tests to be relatively free from erosion tendencies and no damaging erosion was expected there. As a precaution, however, because of the fine-textured friable material composing the channel, rock riprap was used in the prototype channel bottom. No riprap was used in the model tests.

Before the run-off in the Spring of 1950, cross sections had been taken in the prototype channel on May 31, 1949. Following the 1950 run-off, cross sections were again taken on June 15, 1950. Cross sections for both dates at a station located just downstream from the end sill and at a station 50 feet downstream from the sill are shown in Figure 28. These typical sections show the maximum erosion depth to be less than 12 inches. Close to the end sill there

is no significant erosion. Using all the cross sections taken, see Figures 28 and 29, calculations made to determine the volume of material moved indicated that less than 20 cubic yards of material was removed from the channel bottom during the entire run-off.

Conversely, the channel banks, despite their riprap cover, were eroded to a greater degree. The riprap, however, had been placed in a thin layer, was not well graded as to size, and in places the earth banks could be seen between the individual rocks. Swells were observed to rise over local areas and penetrate very easily into the large voids. When the water receded some of the earth was removed from behind the riprap. This was evidenced by the darker, earth-colored water which could be seen adjacent to the riprap. After several days of operation the riprap had slumped and the earth banks had caved as shown in Figure 30. In spite of the apparent damage to the banks the riprap still continued to provide a good measure of protection against further cutting.

The bank damage was not caused primarily by waves of the ordinary variety, since these were only a matter of inches in over-all height, but rather by swells caused by surges in the hydraulic jump. The model stilling basin had been equipped with baffle piers and chute blocks to reduce the over-all length of the stilling basin and decrease its cost. It had been noted during these and other model tests that when a hydraulic jump is reduced in length by the use of artificial devices such as baffle piers that the jump becomes more stable in most respects, but does exhibit a tendency to produce the swells discussed above. The swells are considered the lesser of the evils, however, and are not impossible to cope with. With a base layer of gravel and successive layers of larger and larger rock there probably would have been no damage.

Tail-water Elevations

The topography in the model extended only a short distance downstream from the stilling basin into the excavated channel and did not include any portion of the Heart River, see Figure 9. Tail-water elevations were set by means of an adjustable tail gate located at the end of the model using a computed curve, tail-water elevation versus discharge, for the Heart River. The excavated channel was designed so that the tail-water elevation to be expected would be essentially the same as that to be expected in the river channel. The tail-water curve used in the model tests and shown in Figure 31 was computed for a point located 200 feet downstream from the axis of the dam in the Heart River.

During the prototype operation it was readily apparent to the unaided eye that the tail-water elevation in the river was considerably lower than that in the excavated channel. Water entering the river from the channel had a steep surface slope and a much higher velocity than anticipated, see Figure 32. Observations, however, were not sufficient to establish whether the tail water in the channel was too high or that in the river too low. Levels were run, consequently, to determine the tail-water elevation at four separated points for five different discharges. The location of these points, together with the tail-water elevation and discharge are shown in Figure 31, plotted below the tail-water curve used in the model tests. These data show the computed tail-water curve to be 2.3 feet higher at 1,000 second feet and 4.1 feet higher at 3,600 second feet than the actual measured points in the Heart River. Tail-water elevations measured in the excavated channel more nearly coincided with the computed curve, but at 3,600 second feet the elevation at Point C, Figure 31, taken in a quiet area adjacent to the wing wall at the end of the apron, was 1 foot below

the computed curve. Elevations obtained from water-surface profiles taken along the basin center line agreed with the computed curve, but only because they included the boil height at the end of the apron which was slightly higher than the adjacent tail water.

The model stilling basin was tested to determine the permissible reduction in tail water before the jump was swept off the apron. In the model it was possible to lower the tail water only 3 to 4 feet before the jump was swept out for the maximum discharge of 5,600 second feet. Since the tail-water elevation in the Heart River is 4.1 feet lower, at a discharge of 3,600 second feet, than the computed tail water, it is imperative that a close watch be kept on the excavated channel to prevent damage which might lower the tail water to the level of the Heart River. If this should happen the jump will, beyond a doubt, sweep out and the apron will operate as a flip bucket. Since the structure is not designed for this type of operation, damage could result.

Erosion in the Heart River

The difference in water levels between the excavated channel and the river was the cause of the high-velocity flow entering the Heart River. Water leaving the stilling basin was of relatively low velocity and would not have caused ill effects as it entered the river. The 4-foot difference in elevation, however, caused an increase in velocity which proved to be sufficient to cause considerable damage to the unprotected riverbank downstream. Some of the damage was caused by the direct effects of the current flowing diagonally across the river and cutting into the far or left bank. A great share of the damage, however, was caused by a large induced eddy in the river. This eddy caused an upstream current along the left bank which removed large volumes of material from areas considerably upstream from the point where the main flow impinged on the bank. Although the damage was considerable in extent, it had no ill effects on the structure or its operation. Riprap placed in the eroded area will prove of value, however, since the damage will become greater with each successive run-off and the end result is difficult to predict. The bank damage is illustrated in Figure 33. A comparison of Figures 23 and 34 shows the extent of the bank damage which occurred between the start of the run-off and May 5, 1950.

Inspection of Structure Following 1950 and 1951 Floods

An inspection of the spillway conduit was made following the 1950 flood and again following the 1951 flood. Certain findings are of interest and are discussed in the following paragraphs.

The conduit inspection, in both instances, revealed that the concrete was in excellent condition with the exception of four small areas located in the 90° bend. Following the 1950 flood, plaster casts were made of the two most prominent areas. The largest area is about the size of a man's hand and by actual measurement has a maximum depth of erosion of 3/4 inch, Figure 35. The smaller area shows a maximum depth of 1/2 inch. The surface shown in Figure 35 was molded in sponge rubber against the plaster casts made in the field and are therefore an exact replica of the tunnel surface following the 1950 flood.

The eroded areas are located near the invert and near the bottom of the 90° bend. Construction timbers or ice falling into the shaft could have, by impact, caused the surface damage shown. Persons who have viewed the rubber casts have been of the unanimous opinion that the damage was not started by cavitation.

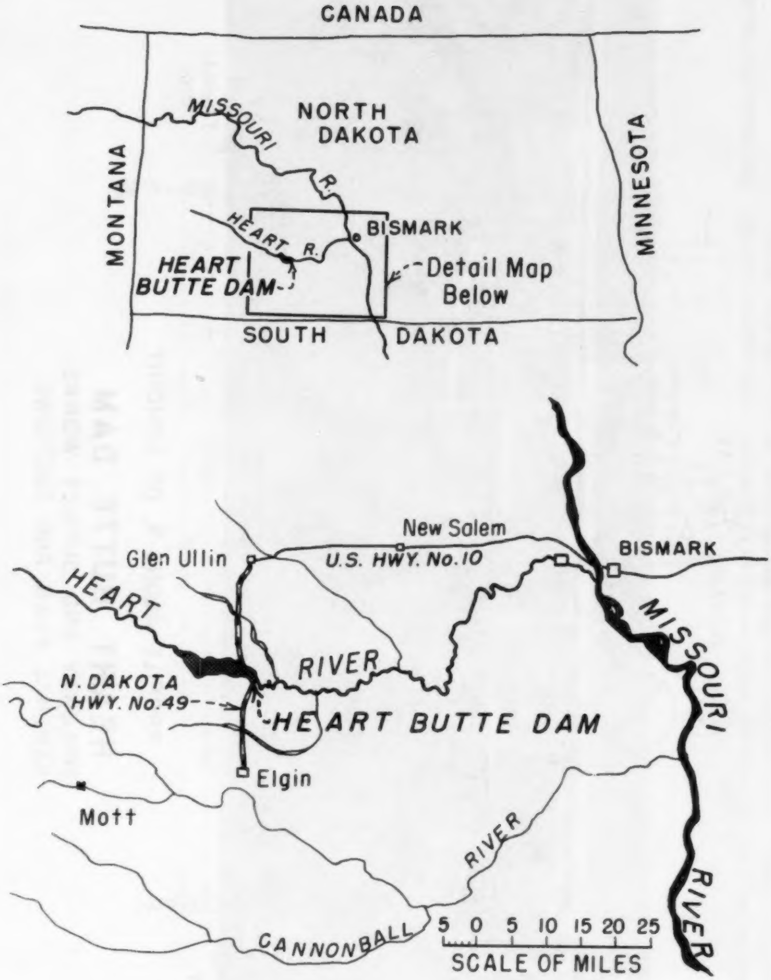
Following the 1951 flood, these areas were again noted. "There did not appear to have been any marked change in these areas as a result of the 1951 spring floods." "No repairs were believed necessary."

Inspection of the riprap downstream from the stilling basin, following the 1950 flood, indicated that repairs would be advisable. The slumped riprap in the channel immediately downstream from the stilling basin structure was repaired in May and June 1950. Gravel backfill was placed on the slopes to bring them to grade and rock replaced over the gravel. The erosion of the riverbank just downstream from the end of the riprapped channel was sloped and covered with gravel and rock.

Acknowledgement

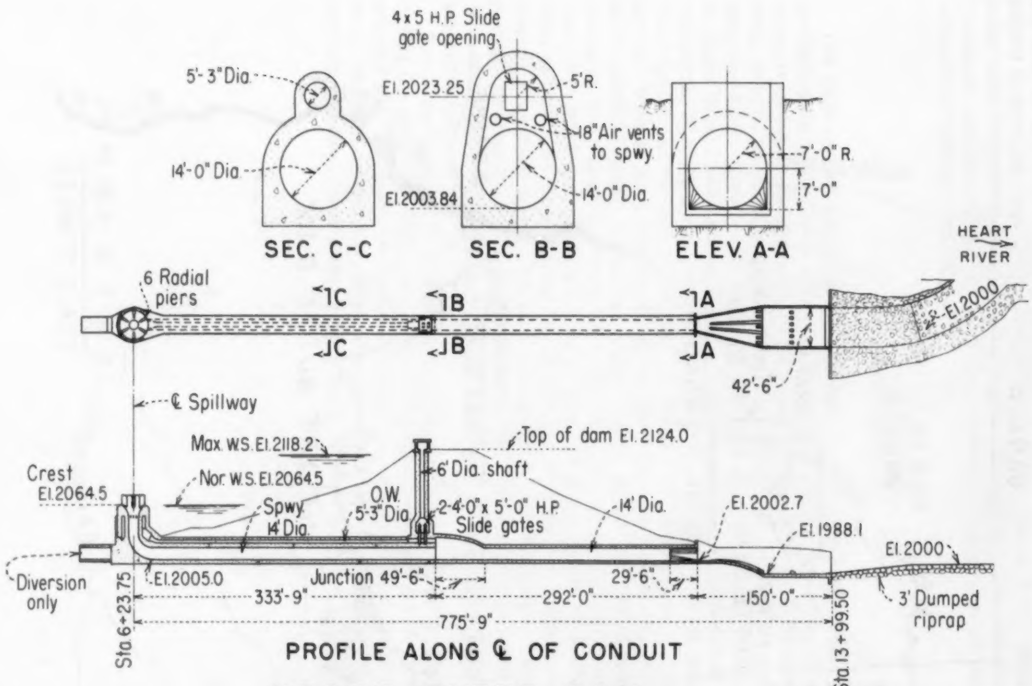
The 1950 run-off occurred without warning at a site located 700 miles from the Hydraulic Laboratory. General flooding of a large area outside of the area protected by the dam caused the road to the site to be flooded and access to the project was difficult. Prototype test equipment was not available at the dam, and trained personnel were not available to make as many observations as might have been considered ideal. The Bureau of Reclamation office in Bismarck, North Dakota, did, however, perform an excellent job of supplying observers from the Glen Ullin office to record data throughout the run-off period. Without their wholehearted cooperation, this report would not have been possible. Their observers supplied most of the data and photographs included in this report and some of the motion pictures of the spillway in operation. The Hydraulic Laboratory contributed to the program by outlining the tests to be made, by supplying the equipment necessary to make measurements, in working up the data and making the final comparisons, and in preparing this report. The United States Geological Survey cooperated with the Bismarck office in supplying men and equipment to make the river gaging measurements.

The assistance of C. F. Burdg, District Engineer, and B. L. Mendenhall, Acting Construction Engineer, in supplying observers and for their wholehearted spirit of cooperation is acknowledged. W. J. Colson, Head of Design Unit; Philip E. Ehrenhard, Materials Engineer; and John Serungard, Chief Inspector, assisted in obtaining photographs, motion pictures, and data.



HEART BUTTE DAM
LOCATION MAP

FIGURE 2



HEART BUTTE DAM
SPILLWAY AND OUTLET WORKS
GENERAL PLAN AND SECTIONS

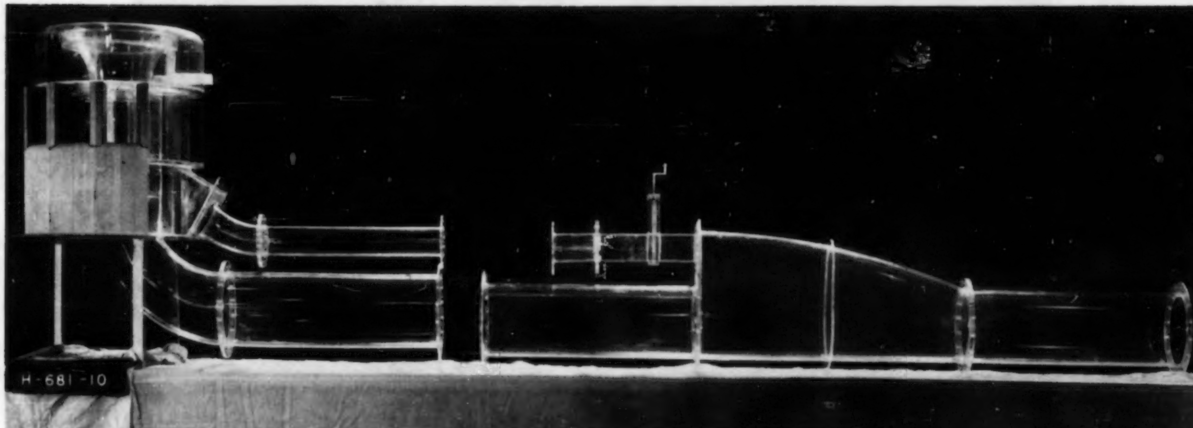


Fig. 3

The essential parts of the spillway and outlet works were modeled in transparent plastic. The intake for the outlet works encircles the spillway shaft. Outlet works discharge is controlled by the slide gate shown.

HEART BUTTE DAM - Spillway and Outlet Works Model, Scale 1:21.5

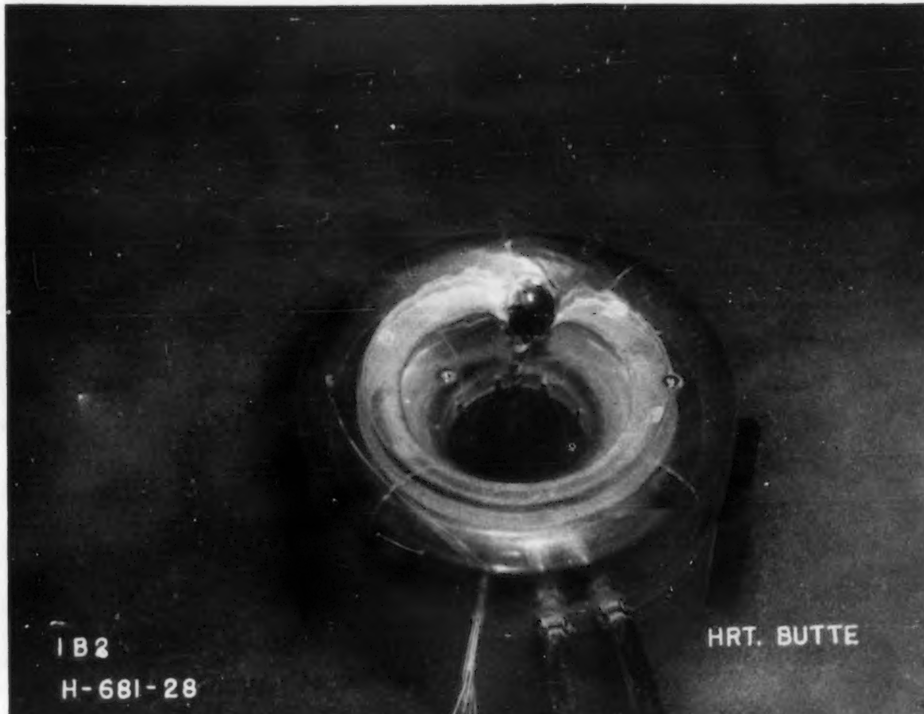
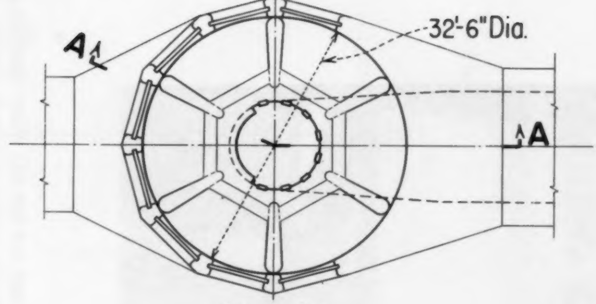


Fig. 4

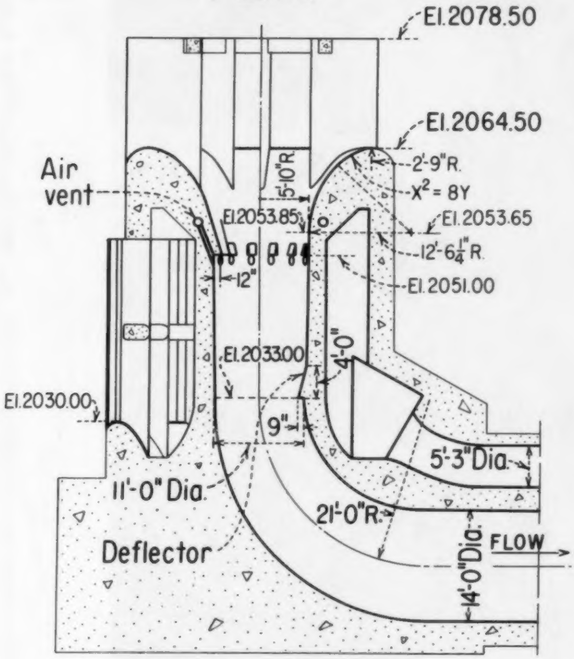
A violent vortex was formed in the spillway for heads above the submergence point. The tail of the vortex extended down into the horizontal tunnel.

HEART BUTTE DAM - Morning-glory Spillway Model Tests—Discharge 3,750 cfs

FIGURE 5



PLAN



SECTION A-A

HEART BUTTE DAM
SPILLWAY AND OUTLET WORKS
ENTRANCE DETAILS

301

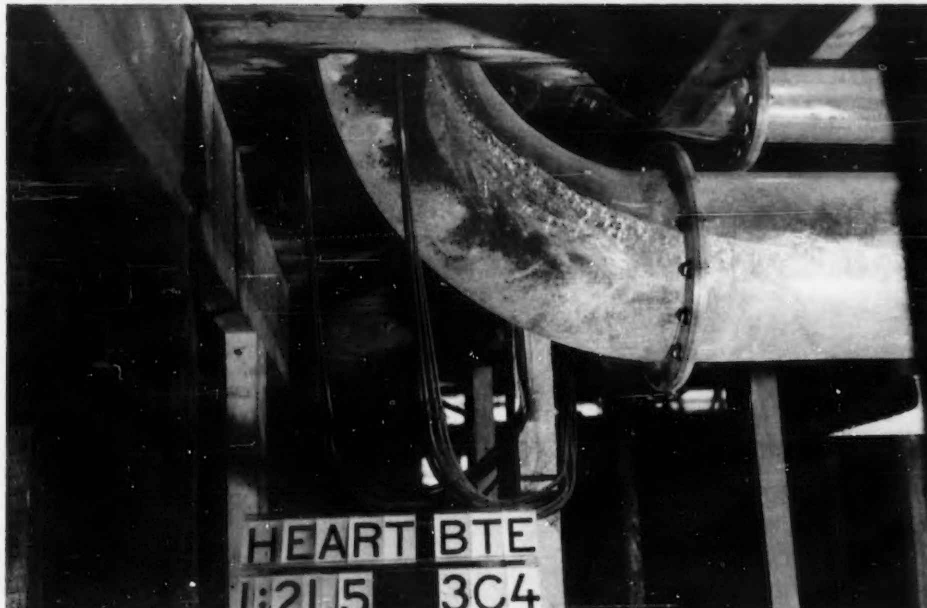
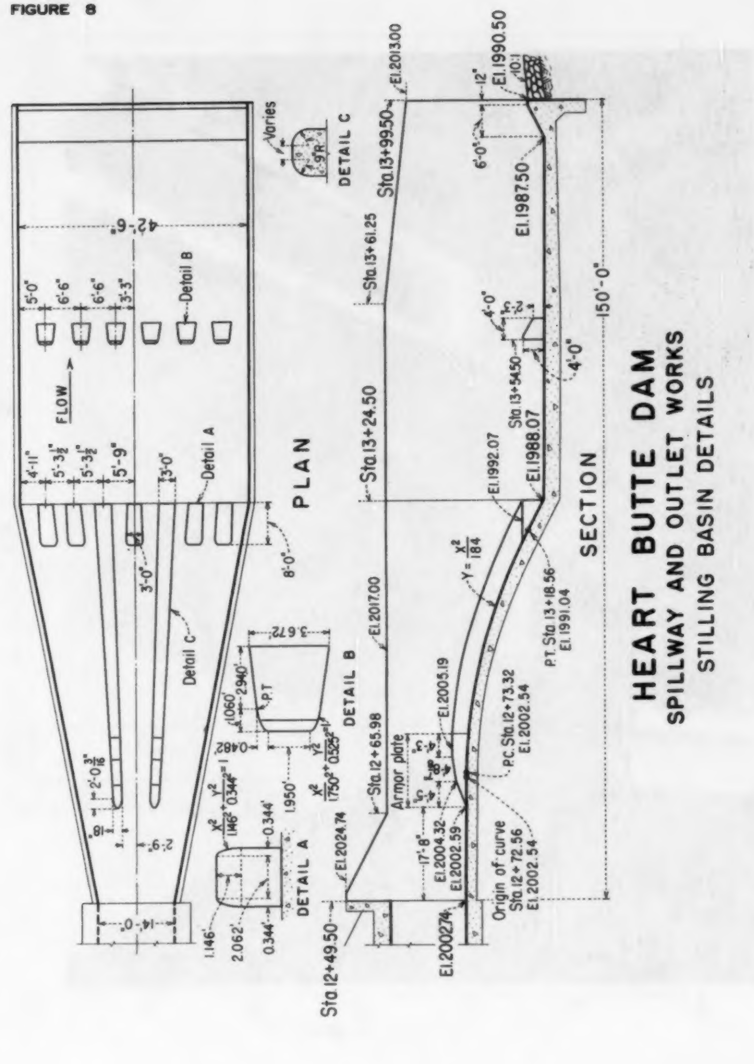


Fig. 6

Flow in the vertical bend for a discharge of 3,750 cfs. Note the smooth flow in the bend and the flat water surface produced by the deflector at the base of the vertical shaft. 1/15,000-second exposure shows air entering flow in bursts. To the eye air flow appeared continuous.

HEART BUTTE DAM - Hydraulic Model (Vertical Bend) Tests

FIGURE 8



HEART BUTTE DAM
SPILLWAY AND OUTLET WORKS
STILLING BASIN DETAILS

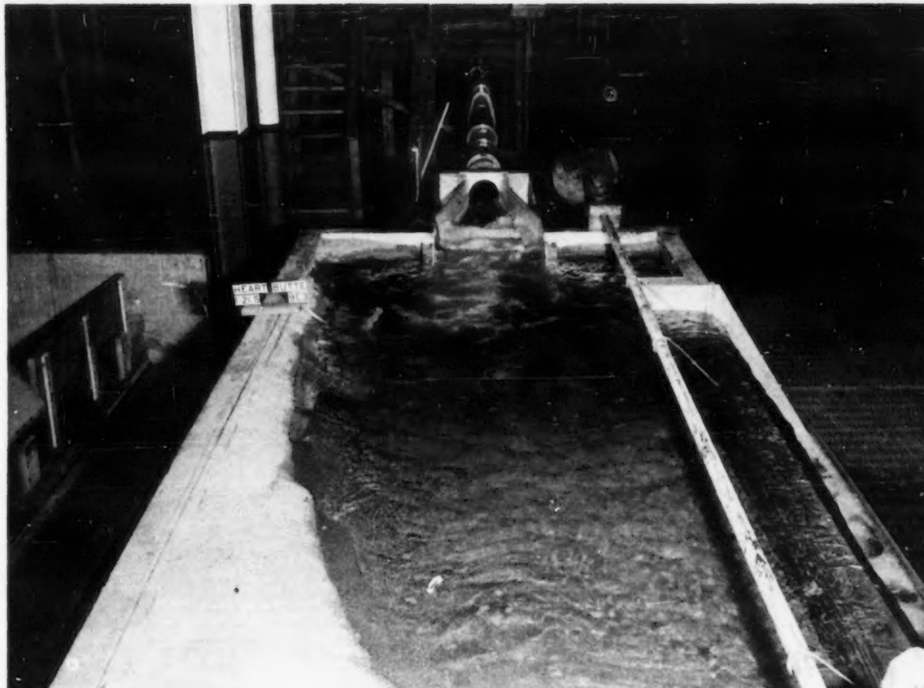


Fig. 9

The recommended basin, shown in operation, was reduced to minimum dimensions consistent with acceptable performance.

HEART BUTTE DAM - Stilling Basin Model Tests. Discharge 5,600 cfs—Tail-water Elevation 2012

FIGURE 10

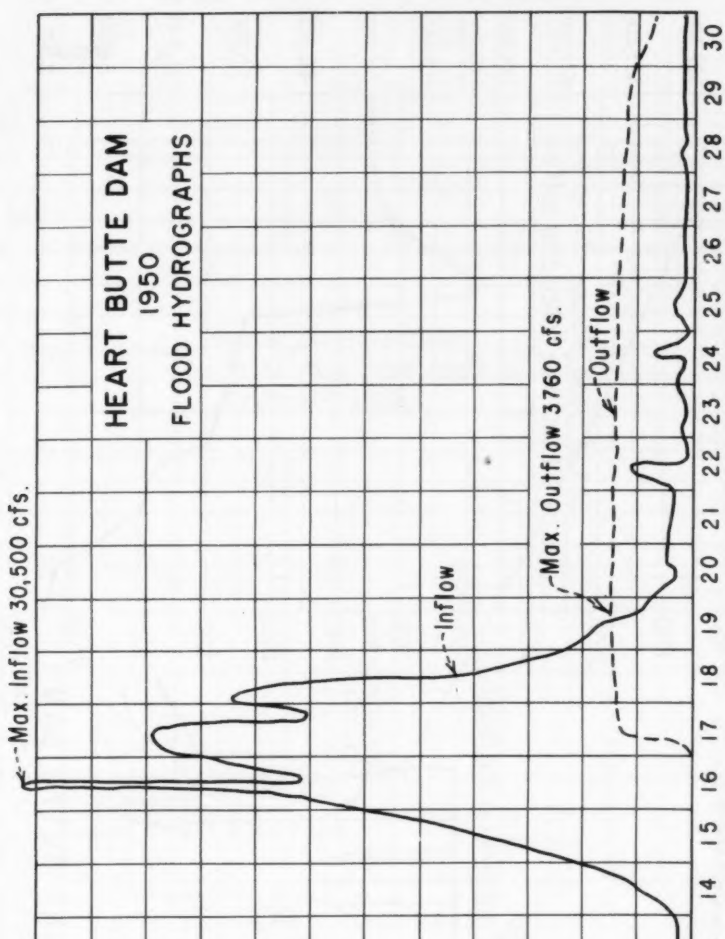
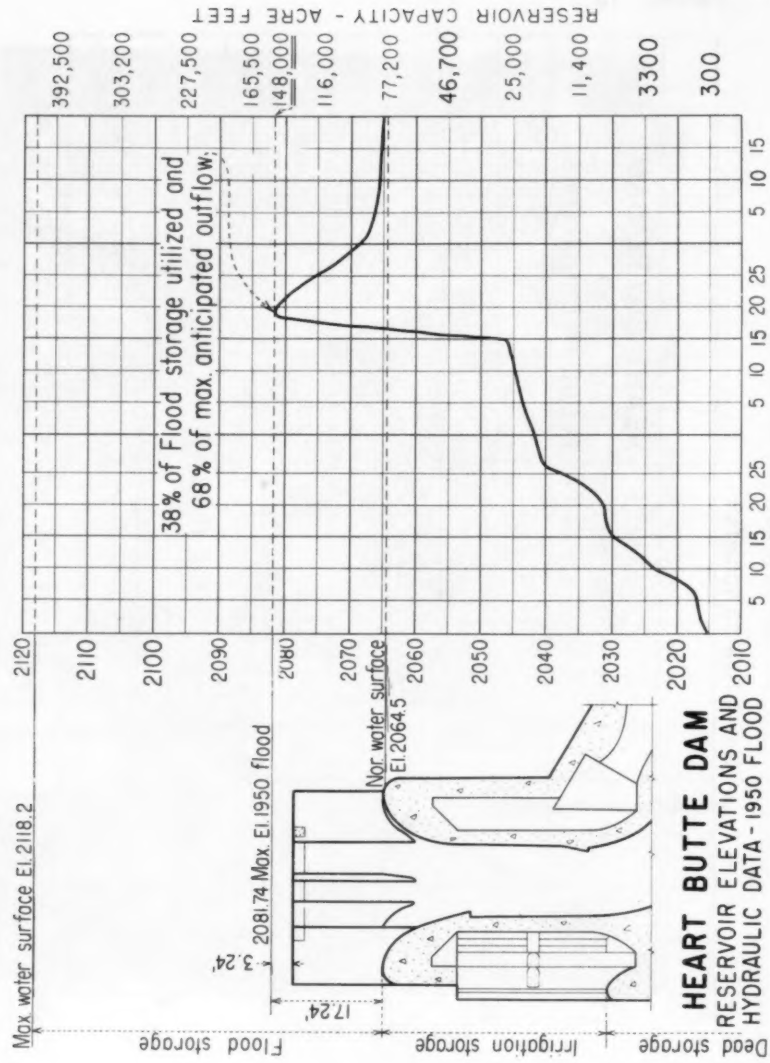
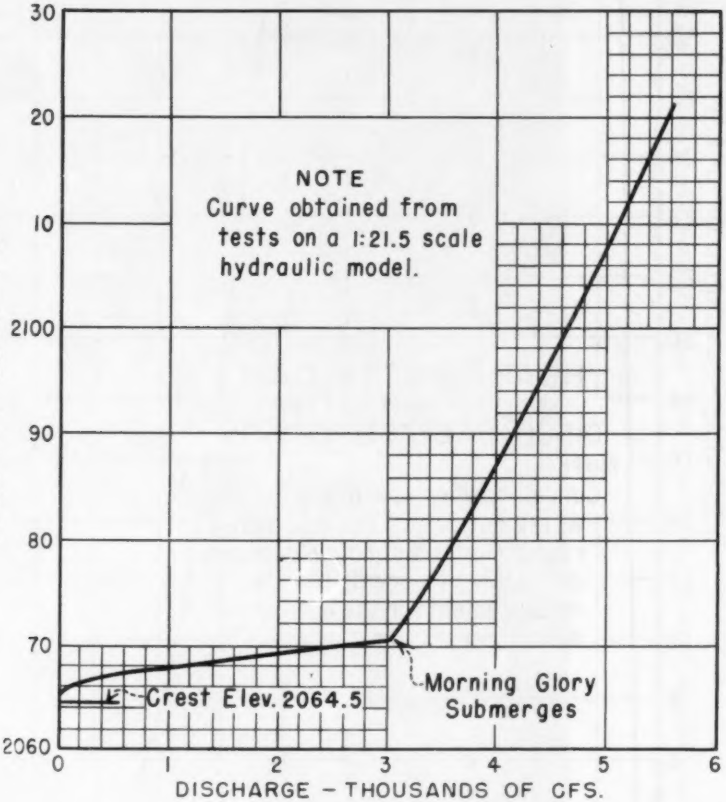

HEART BUTTE DAM
1950
FLOOD HYDROGRAPHS
Max. Inflow 30,500 cfs.
Max. Outflow 3760 cfs.
Inflow
Outflow
FLOW-1000-CFS
30
25
20
15
10
5
APRIL
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30

FIGURE 11

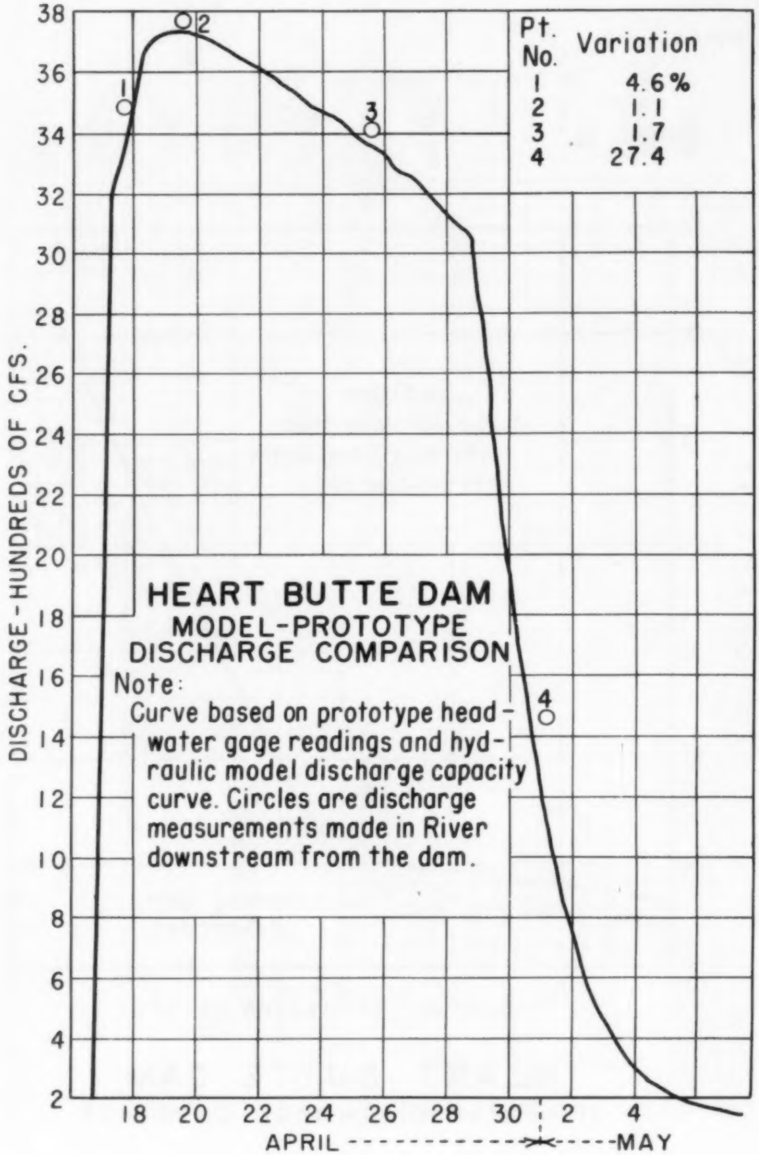


HEART BUTTE DAM
RESERVOIR ELEVATIONS AND HYDRAULIC DATA - 1950 FLOOD

FIGURE 12.



HEART BUTTE DAM
SPILLWAY DISCHARGE CAPACITY


38
36
34
32
30
28
26
24
22
20
18
16
14
12
10
8
6
4
2
DISCHARGE - HUNDREDS OF CFS.
18
20
22
24
26
28
30
2
4
APRIL
MAY
1
2
3
4
Pt. No.
Variation
1 4.6%
2 1.1
3 1.7
4 27.4
HEART BUTTE DAM
MODEL-PROTOTYPE
DISCHARGE COMPARISON
Note:
Curve based on prototype head-water gage readings and hydraulic model discharge capacity curve. Circles are discharge measurements made in River downstream from the dam.

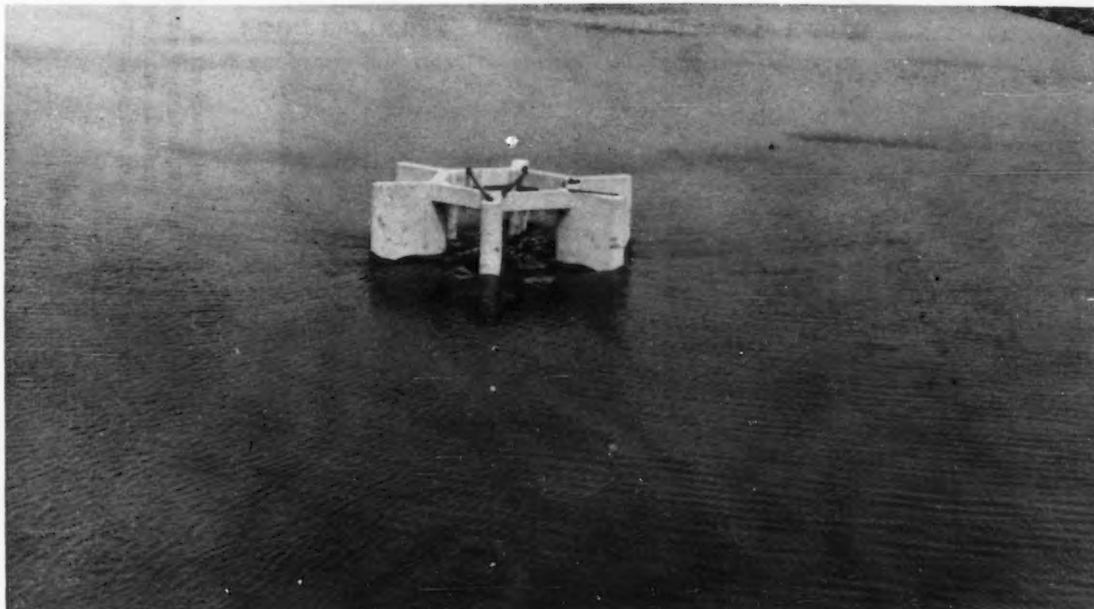


Fig. 18

On April 28, 1950, the reservoir was at elevation 2071, or about 0.7 foot above the point where the flow changes from free to submerged. Action inside the morning-glory is very mild. Note flow lines visible on water surface.

HEART BUTTE DAM - Spillway Operation—Discharge 3,090 cfs

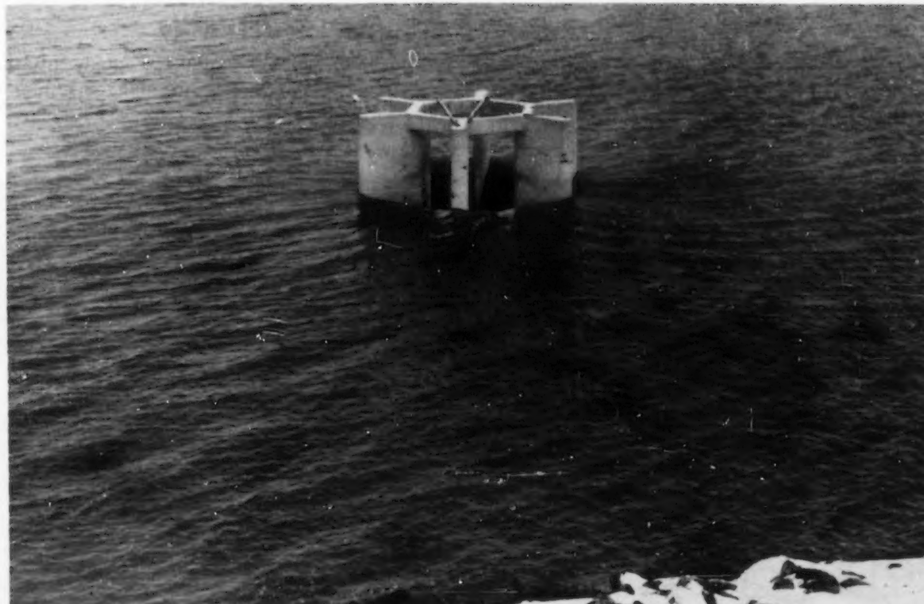


Fig. 19

The spillway was discharging freely on April 30, 1950, with reservoir elevation 2068.6.

HEART BUTTE DAM - Spillway Operation—Discharge 1,600 cfs

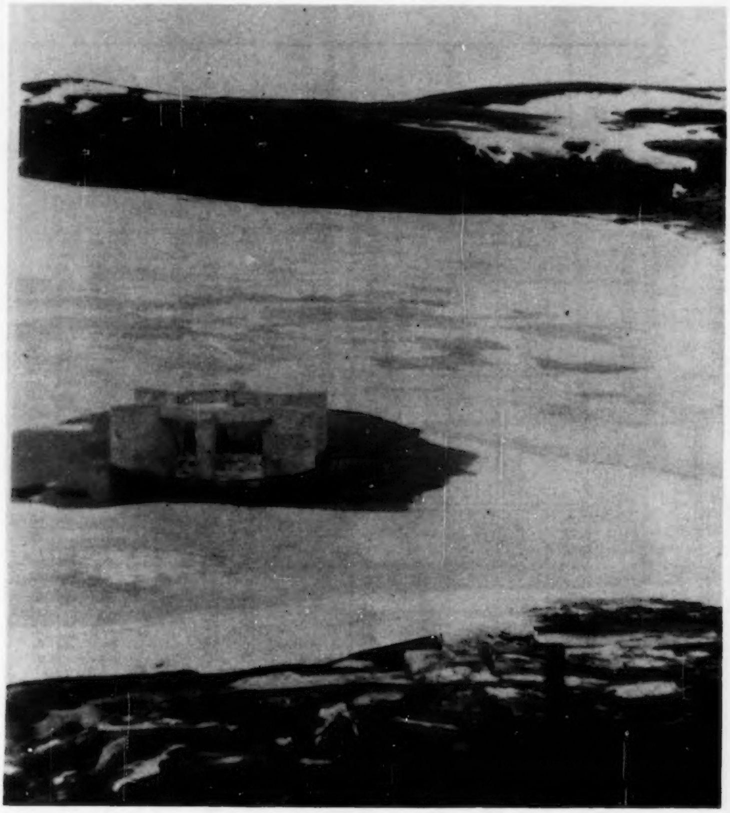
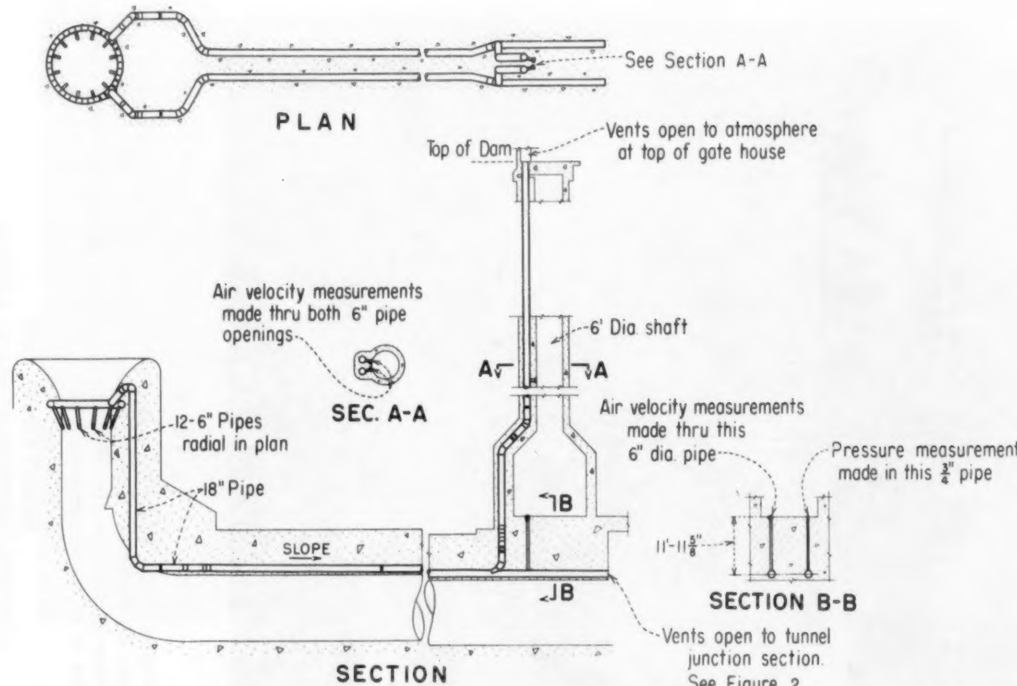


Fig. 20

In the Spring of 1951 the reservoir rose to elevation 2075.0. This photo, taken on March 27, shows the reservoir at elevation 2070.0, or about 0.2 foot below the submergence point. In February the ice was 36 inches thick but no difficulties due to ice were encountered.

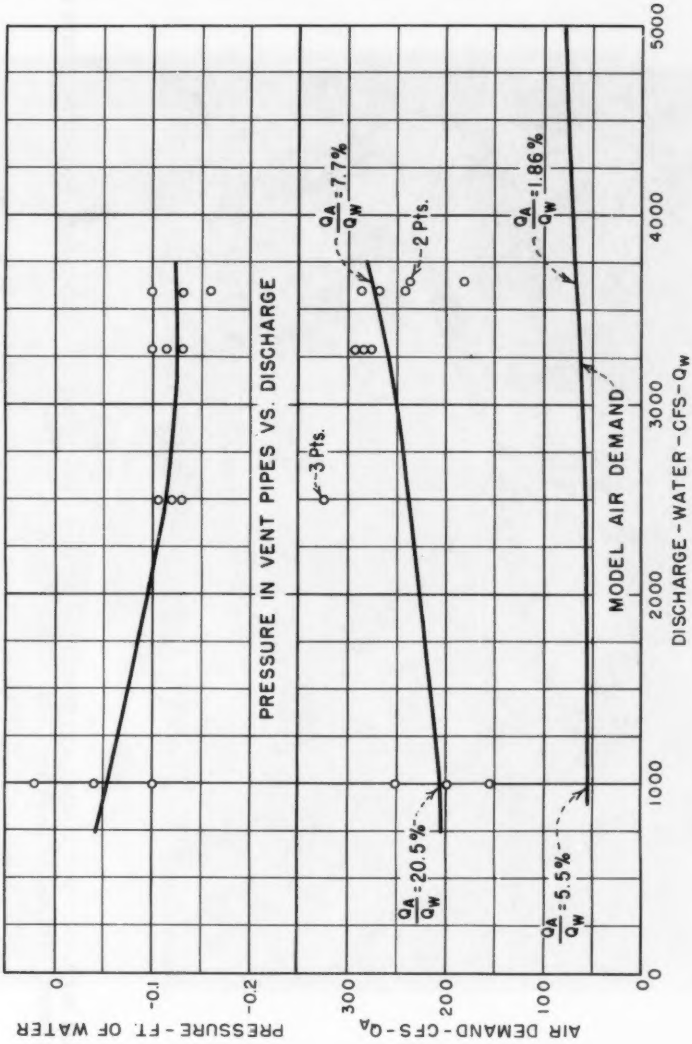
HEART BUTTE DAM
Spillway Operation—Discharge 2,650 cfs



HEART BUTTE DAM
SPILLWAY AIR VENT SYSTEM

FIGURE 21

FIGURE 22



HEART BUTTE DAM
SPILLWAY AIR DEMAND VS. DISCHARGE

Fig. 23

Stilling basin, excavated channel, and Heart River as seen from the top of the dam. Basin was very effective in dissipating energy. Outflow is about 58 percent of capacity. April 17, 1950.

HEART BUTTE DAM - Stilling Basin Performance—Discharge 3,250 cfs

Fig. 24

The profile of the hydraulic jump is indicated along the stilling basin wall. Flow leaving the basin is smooth and uniform.

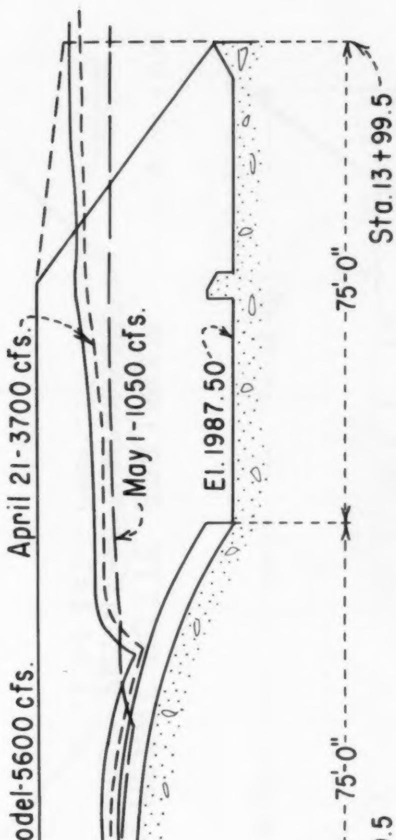
HEART BUTTE DAM - Stilling Basin Performance—Discharge 3,600 cfs

Fig. 25

Flow leaving the basin had no choppy waves but did have mild swells and boils.

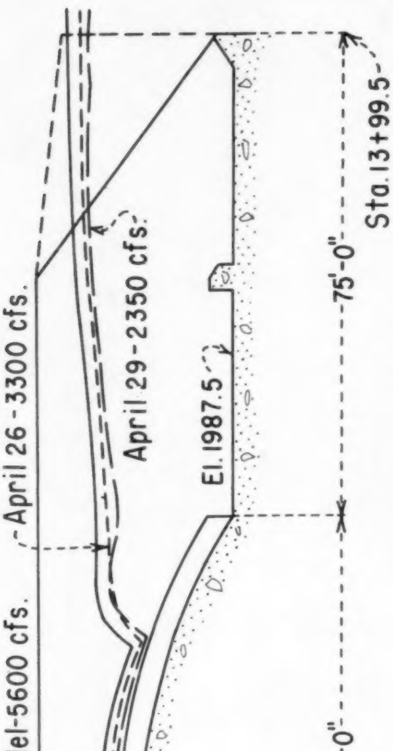
HEART BUTTE DAM - Stilling Basin Performance—Discharge 3,600 cfs

FIGURE 26



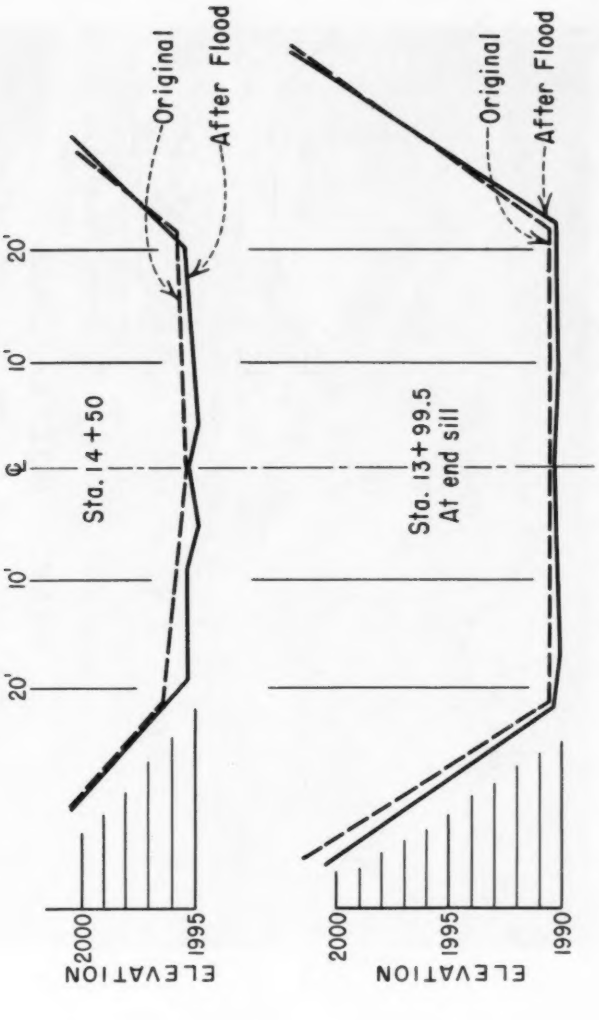
HEART BUTTE
MODEL-PROTOTYPE COMPARISON
STILLING BASIN PROFILES

FIGURE 27



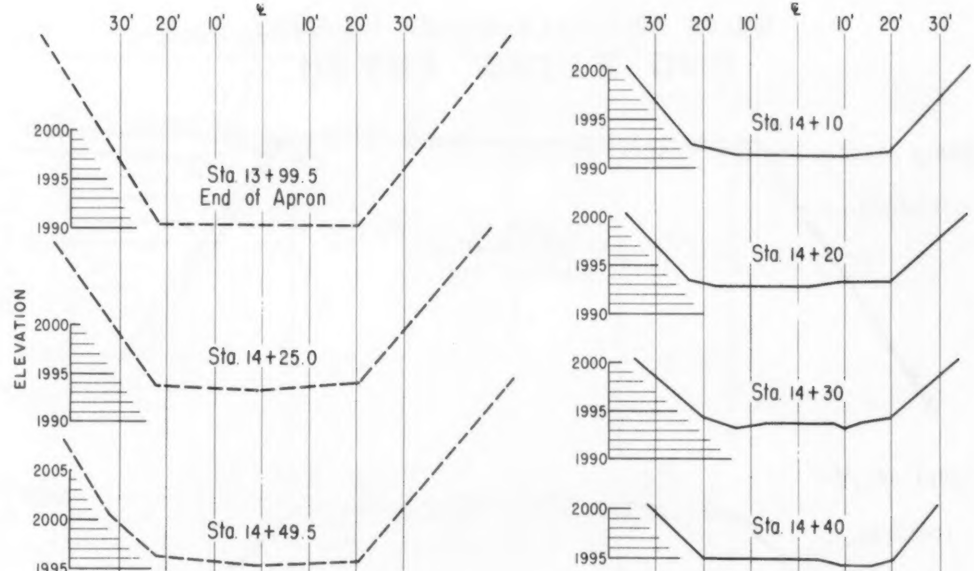
HEART BUTTE SPILLWAY
MODEL PROTOTYPE COMPARISON
STILLING BASIN PROFILES

FIGURE 28



HEART BUTTE DAM
EROSION BELOW STILLING BASIN

FIGURE 29



SECTIONS TAKEN MAY 31, 1949
(Original)

SECTIONS TAKEN JUNE 15, 1950
(After flood)

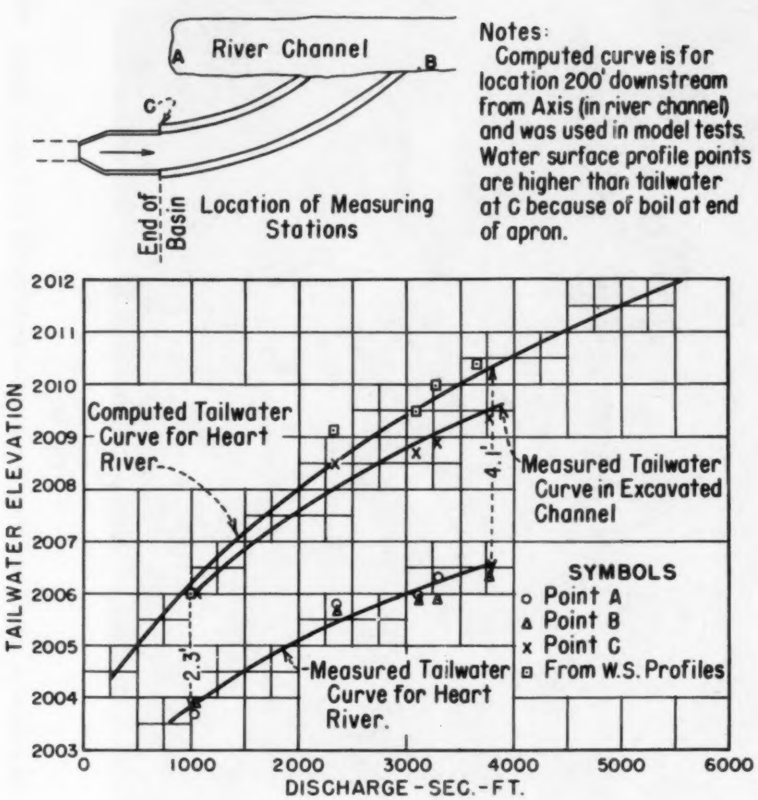
HEART BUTTE DAM

CROSS - SECTIONS BELOW STILLING BASIN

Fig. 30

Loss of bank material was caused by swells removing fine material from behind the coarse riprap. Note man standing on the riprap.

HEART BUTTE DAM - Erosion of Excavated Channel Banks



HEART BUTTE DAM
COMPARISON OF MEASURED AND COMPUTED TAILWATER ELEVATION

Flow entering the river from the excavated river channel was accelerated by the difference in elevation apparent in the photograph. At 3,700 cfs the difference was about 4 feet.

HEART BUTTE DAM - Flow Conditions—Junction of Channel and River

Fig. 33

After flood had receded the loss of bank material opposite the excavated channel was readily apparent.

HEART BUTTE DAM - Riverbank Erosion

Fig. 34

Over-all view after flood had passed shows extent of downstream bank erosion on May 5, 1950. Note bars formed from eroded material. Compare with Figure 23.

HEART BUTTE DAM - Downstream Bank Erosion

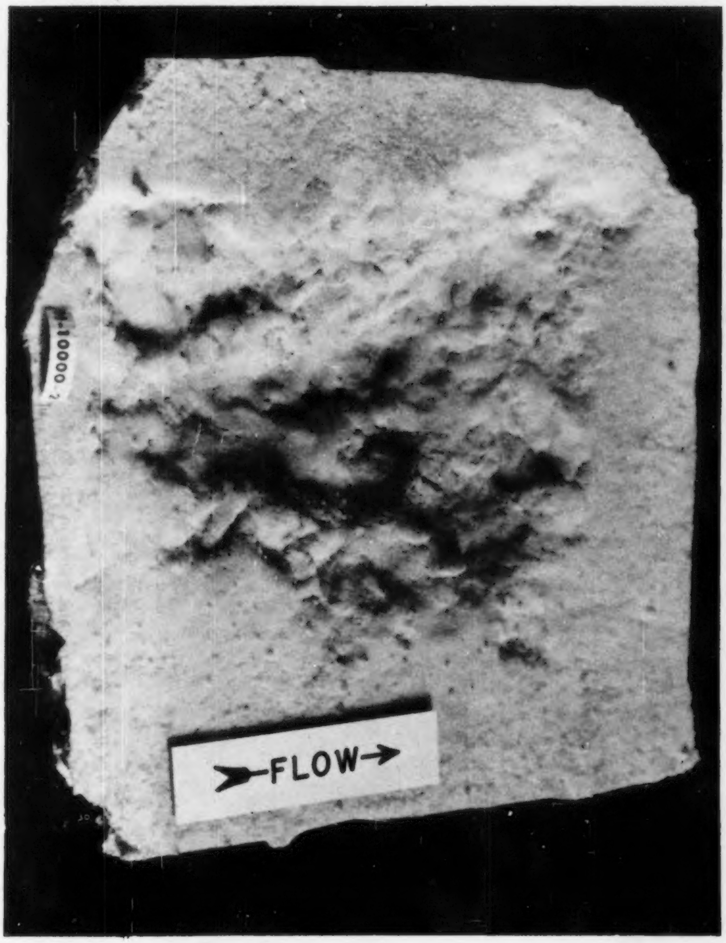


Fig. 35

One of the four eroded areas found in 90° bend after the 1950 flood had passed. This area, shown full size, has the deepest erosion, 3/4 inch.

HEART BUTTE DAM

Eroded Area in 90° Bend—"Upper Right"